

# PROCEEDINGS

## AMERICAN SOCIETY OF CIVIL ENGINEERS

JULY, 1955



### FIELD VANE SHEAR TESTS OF SENSITIVE COHESIVE SOILS

by Hamilton Gray, M. ASCE

### SOIL MECHANICS AND FOUNDATIONS DIVISION

*{Discussion open until November 1, 1955}*

*Copyright 1955 by the AMERICAN SOCIETY OF CIVIL ENGINEERS  
Printed in the United States of America*

**Headquarters of the Society**  
33 W. 39th St.  
New York 18, N. Y.

PRICE \$0.50 PER COPY

## THIS PAPER

--represents an effort by the Society to deliver technical data direct from the author to the reader with the greatest possible speed. To this end, it has had none of the usual editing required in more formal publication procedures.

Readers are invited to submit discussion applying to current papers. For this paper the final date on which a discussion should reach the Manager of Technical Publications appears on the front cover.

Those who are planning papers or discussions for "Proceedings" will expedite Division and Committee action measurably by first studying "Publication Procedure for Technical Papers" (Proceedings Paper No. 290). For free copies of this Paper—describing style, content, and format—address the Manager, Technical Publications, ASCE.

Reprints from this publication may be made on condition that the full title of paper, name of author, page reference, and date of publication by the Society are given.

The Society is not responsible for any statement made or opinion expressed in its publications.

This paper was published at 1745 S. State Street, Ann Arbor, Mich., by the American Society of Civil Engineers. Editorial and General Offices are at 33 West Thirty-ninth Street, New York 18, N. Y.

## FIELD VANE SHEAR TESTS OF SENSITIVE COHESIVE SOILS

Hamilton Gray,<sup>1</sup> M. ASCE

### INTRODUCTION

During the past few years a considerable amount of effort has been expended in certain parts of Europe and, to a lesser extent, localities in the United States in appraising the significance of values of soil shear strength which are based upon test methods which do not require the extraction of soil samples from the ground. The results of such methods of testing have not been widely publicized in this country, but it is believed that as the results become better known these new methods may in many localities for reasons of reliability and economy logically replace the ones conventionally used.

In spite of successful efforts<sup>2</sup> to develop techniques intended to minimize the disturbance of sensitive cohesive soils in sampling operations there is available much evidence indicating that the best of so-called "undisturbed" samples<sup>3</sup> of many clays do not reveal with desirable accuracy the actual mechanical properties of the various soil layers from which they are extracted. It is generally conceded that the stress changes associated with removal of soil samples from the ground must adversely affect the results of tests made on such samples and that distortions of the mineral skeleton will introduce relatively important changes in behavior under stress. Highly sensitive clays, and the accompanying difficulty in preserving the soil structure, have been encountered chiefly in alluvial, lacustrine, and marine deposits, while in many regions mantled by residual soils, the sensitivity of the materials does not appear to be sufficient to warrant much concern with regard to possible sample disturbances. The purpose of this paper is to relate experiences obtained in exploring deposits of glacial and marine clays in southern Maine. While the results and conclusions are believed applicable to many similar deposits in the northern part of the United States they would not be equally significant for residual soils.

### Sensitivity of Clays

Certain sedimentary clays or silt-clays possess extremely high degrees of "sensitivity." The sensitivity of a soil has been defined<sup>4</sup> in terms of the ratio of the unconfined compressive strength of an undisturbed sample of "good" quality to the compressive strength of the same material after having been

1. Prof., Civ. Eng., Univ. of Maine, Orono, Me., & Soils Engineer, Maine Highway Commission.
2. Hvorslev, M. J., Subsurface Exploration & Sampling of Soils for Civil Engineering Purposes. Engineering Foundation.
3. Also known as "practical undisturbed samples."
4. Terzaghi, K. & Peck, R. B., Soil Mechanics In Engineering Practice, John Wiley & Sons.

thoroughly remoulded at unchanged water content. This sensitivity so defined is found to vary from values slightly greater than unity to as much as 30 or 40. In some cases where the natural water content exceeds the liquid limit a prism of remoulded material will distort appreciably under its own weight. Therefore, the sensitivity can hardly be measured in the customary manner and may even be said to approach infinity. Since partial remoulding and hence some weakening of a soil may occur as a result of sampling operations, it should be realized that such a definition of sensitivity is based upon the natural consistency of such samples as are available and that frequently if the sensitivity could be referred to the strength of the natural deposit even greater values would result. Highly sensitive materials, which are fairly common in the northern half of the United States, can be detected by the obvious contrast between consistency in the "undisturbed" and remoulded states which becomes evident when the undisturbed soil is manipulated with the fingers. The sensitivity is dependent upon a) the extent to which the natural moisture content differs from the liquid limit, and upon b) the magnitude of the preconsolidation load which strongly influences the undisturbed strength. Thus, the higher the moisture content relative to the liquid limit, the weaker the remoulded material, and the higher the preconsolidation load, the stronger the undisturbed material. Consequently, if the natural water content of a highly preconsolidated soil is well above the liquid limit the material will be excessively sensitive, whereas if the natural moisture content approaches the plastic limit when the preconsolidation load is small, the sensitivity will be correspondingly small. In spite of the fact that certain clays have been compressed under geologic loads exceeding one ton/sq.ft. and, consequently, have acquired strengths of the order of  $1/4$  to  $1/2$  ton/sq.ft. or more, the natural void ratios may be so high that the remoulded materials exhibit an almost fluid consistency and, consequently, possess extreme sensitivity.

In engineering practice it is recognized that further compression, involving a reduction in void ratio, of such soils will increase their shearing strength but that although such compression or decrease in void ratio may be produced through appropriate loading, it cannot materialize immediately upon the application of a load because of the consolidation characteristics of the soil. In other words, unless special provisions are made to allow compression of the material it is necessary to rely upon only the strength which the material possesses in its natural condition prior to construction operations. This strength can be determined by applying shearing stresses in such a manner that the soil has no opportunity to compress and thus acquire additional strength during the application of the shearing stresses. The type of test which reveals the existing strength of a clay soil is commonly referred to as "quick undrained," or merely "quick" since speed of testing is assumed to preclude effective drainage.

#### Strength Determinations

In the past it has been common practice to determine such strengths by means of unconfined compression tests since such tests can be made more rapidly and conveniently than the conventional type of direct shear test. Consequently, the compressive strength has been used both as a basis for measuring the effect of sample disturbance and for securing data for engineering studies of foundation conditions. Interest in the use of a method of determining the strength of soil in its natural bed was stimulated by the following observations.

It is generally believed that the soil adjacent to the wall of any type of sampling tube is weakened by frictional drag and consequent relatively severe distortion adjacent to the tube wall. In fact data have been published showing that the compressive strength or consistency of miniature compression specimens located at varying distances from the wall of a sampling tube increases as the distance from the wall increases.<sup>5</sup> In other words, the center of the sample is demonstrated to have been less damaged than the periphery by the sampling operation, thus confirming the visual evidence of excessive peripheral distortion which is often observed. Therefore, if a prism measuring, for example, two (2) inches square in cross-section is carefully trimmed from the innermost portion of a three and one-half (3-1/2) inch diameter soil sample, it is logically assumed that the weakest material has been trimmed away and discarded leaving relatively undisturbed soil in the prism to furnish the basis for measuring soil strength. However, a comparison of the compressive strengths of certain "undisturbed" samples secured in 3-1/2" O.D. seamless tubing of #16 gauge wall thickness indicated that the strengths of compression specimens having a diameter equal to that of the inside of the tubing were not appreciably different from strengths of specimens which had been carefully trimmed with a wire saw from "tube size" to a cross-section measuring two inches square. The test results thus indicated that the elimination of the peripheral material did not change the results of compression tests of many sensitive soils and it was, therefore, necessary to conclude that the trimming operation had seriously weakened the two inch by two inch prism. Consequently, the trimming operation appeared to be an unnecessary and time-wasting refinement.

The two inch by two inch prisms were trimmed carefully by hand with an ordinary wire saw guided by a miter box. It is possible that if such a saw were operated in conjunction with some high frequency vibrator less disturbance would occur during the trimming operation. However, there is ordinarily required a minimum of handling of the trimmed prism which may in itself be just as damaging as the trimming operation. At any rate it is apparent that if, as is generally conceded, peripheral weakening occurs during sampling, some better means than trimming away peripheral material should be adopted if realistic strength values are to be consistently obtained for highly sensitive clays. The mere fact that much depends upon the care exercised by the person trimming and otherwise preparing the prisms is sufficient cause to seek a means of eliminating the "personal touch."

#### Minimizing the Effect of Sensitivity on Test Results

The logical way of eliminating the disturbance which accompanies the trimming as well as the distortion of that portion of a sample which is adjacent to the wall of the sampling tube is to determine the strength of the central portion of the sample in some manner which does not require that this central portion be either trimmed or removed from the sampling tube. To accomplish this a small four-bladed vane having a diameter of one inch was constructed and placed in such a position that it could be inserted in the end of a tube sample and then rotated on anti-friction bearings by means of a small torque wrench. Figure #1 shows details of such a "miniature" vane and of its operation. When the results of such vane tests were compared with

5. Burmister, D.M., A Method for Determining the Representative Character of Undisturbed Samples, etc. Proceedings, First International Conference on Soil Mechanics, 1936.

the results of unconfined compression tests performed upon material taken from an adjacent portion of the tube sample, either above or below the section which was subjected to the vane test, very consistent differences appeared. The vane strength is with rare exceptions, usually attributable to heterogeneity of the sample, substantially greater than half the compressive strength. This confirmed the belief that the separation of the central portion of a tube sample from the peripheral portion together with the other procedures involved in preparing this central prism for a compression test introduced disturbances of sufficient magnitude to seriously depress the results.

It is obvious that this information at best could only imply that the miniature vane test results more nearly approximated the true strength of the natural soil, but that no indication has been obtained as to the accuracy of this approximation. In other words it still was necessary to extrapolate from the results of tests performed on samples in order to estimate the strength of the natural deposit. Any such extrapolation rests on no fully objective considerations, and is therefore unsatisfactory.

There appeared to be no reason why the same testing principles should not be utilized in measuring the soil strength in the field.<sup>6</sup> Figure #2 shows details of larger vanes which can be used to measure the soil strength at the bottom of bore holes two and one-half or four inches in diameter. Such a vane, attached to the lower end of a string of drill rods, is inserted into the undisturbed material beneath the lower edge of the casing at the bottom of a cased drill hole. In order to prevent vertical movement of the vane during the test the drill rods are supported on the top of the casing by means of a combination thrust and radial bearing. The lengths of vanes may be varied in accordance with the anticipated resistance of the soil since the estimated safe loading capacity of each vane is as indicated on figure 2. The vanes are rotated by means of torque wrenches applied at the top of the drill rods. The torque wrenches are likewise varied so that a more sensitive type of wrench is used where the total torque is expected to be small. The performance of each field vane test requires somewhat less time than is needed in taking an undisturbed sample. It appears advisable to devote nearly as much care to the cleaning of the bore hole prior to making the vane test as is desirable prior to obtaining a sample. This care has a twofold purpose:

- 1) To ensure that the material in which the vane is rotated is in no way disturbed.
- 2) To eliminate soil friction from the drill rods above the vane.

The rotation of the vane requires less time than is usually allowed to elapse between driving and extraction of a sample tube, and also once the vane test has been performed, a saving of time can result from the fact that no sealing, marking and transportation of an undisturbed sample may be necessary nor need an unconfined compression test be performed except to secure comparative data. On the other hand, the making of field vane tests does not necessarily eliminate the need for taking seamless tube samples inasmuch as consolidation data may be required, and in any event it is very desirable to secure sufficient material from different elevations in the ground to reveal by visual inspection and classification tests the variability of the subsurface formations.

6. The feasibility of this was suggested by the following account of European experience:

Carlson, Lyman, "Determination in Situ of the Shear Strength of Undisturbed Clay by Means of a Rotating Auger." 1949. Proceedings, 2nd International Conference on Soil Mechanics.

Consequently, the real justification for such field vane tests resides in the greater reliability which the results appear to possess.

### Typical Test Results

Figures 3 and 5 illustrate for two typical borings made on two different projects:

- 1) the soil stratification,
- 2) the elevation at which soil samples and field vane tests were obtained,
- 3) the resistance offered to drilling,
- 4) the natural water contents of the samples, and
- 5) the Atterburg limits.

In both these borings the samples were secured in 3-1/2 inch O.D. seamless tubing of 16 gauge wall thickness and all vane tests were made with the large-diameter vane shown on figure 2. The bulk of the penetrated soil was sufficiently soft so that the sample tubes could be made to penetrate the necessary length of approximately 30 inches rapidly under the static load of the drill rods plus a 300 lb. hammer. Where the soil was too hard to permit penetration under a static load of 600 lbs. applied to the top of the drill rods, the sample tubes were driven by the impact of a 300 lb. hammer falling through a distance of approximately 15 inches upon the top of the drill rods. The number of impacts required to produce 1 ft. of penetration is indicated by the lengths of heavy horizontal bars adjacent to the sample markers on figure 3. With these exceptions it was unnecessary to use dynamic effort to drive the sample tubes. The number of hammer blows required to sink the casing one foot is indicated by an irregular line.

It will be observed that the natural moisture contents shown in figure 3, tend to exceed the corresponding liquid limits, in some instances by a very substantial amount. Thus one requirement for a high degree of sensitivity is generally present in that boring. In the case of figure 5 the water contents in the upper half of the deposit fluctuate erratically because of the varved nature of the soil. The water contents in this zone all appear to lie within the plastic range, whereas in the lower portion of the boring, water contents do not depart as greatly from a mean value and remain consistently greater than the liquid limits. In figure 6, there is evidence of weathering or desiccation in the relatively high strengths of the uppermost part of the soil profile.

Examination of the shear strength data pictured in figures 4 and 6 demonstrates clearly a tendency for the field vane test to yield the greatest shear strengths while the unconfined compression tests produce the least strengths and the miniature vane strengths attain intermediate values. The general trend of strength in any one boring, apart from the influence of desiccation of the surficial material, is to increase with increasing depth. The substantial scattering of individual values from the average trend is, it is believed, attributable to the thinly laminated or varved nature of these deposits. Any single determination of strength may represent the effect of more or less granular material which is present in a predominantly cohesive material. In order to aid in visualizing the variations in strength with depth, which are revealed by the three types of test, straight lines which best fit the data have been determined for figures 4 and 6 by the method of least squares. The graphical representations of the resulting equations appear in these figures.

These results are typical of those obtained from a substantial number of drill holes made in connection with the planning of embankments of varying heights. However, in these other borings less comprehensive data were

usually secured, in that less effort was made to determine the unconfined compressive strengths after vane tests had been made and the vertical intervals between vane tests were greater than in the two borings illustrated herein. The less complete data substantiate those shown on figures 3 to 6 inclusive. The primary consideration in each case was that the embankment not induce shearing displacements in the supporting ground. Settlements constituted a secondary consideration, for if a settlement of moderate magnitude was predicted on the basis of consolidation tests it was generally felt that the displacement could be tolerated and even compensated either by appropriate temporary overloads which would accelerate compression during construction, or by future maintenance.

### Significance of Results

It is but logical to believe that the differences observed between the results of field and laboratory vane tests are caused solely by sampling operations and that the differences between the laboratory vane and unconfined compression results are largely attributable to the preparation of the compression specimens. The first of these differences represents disturbance which occurs before any laboratory testing can be done, and the second type of difference indicates the extent to which laboratory preparation may further injure the structure of a sensitive soil. In comparative tests performed upon remoulded soils, laboratory vane results tended to exceed somewhat the half-magnitudes of corresponding compressive strengths. The discrepancies were not serious and may be attributable to the fact that failure of compression specimens occurred by plastic deformation whereas the vane resistances were associated with a surface of rupture. Inspection of figures 4 and 6, suggests that the greatest strength differences occur towards the bottoms of the drill holes where the natural strengths are higher, but where in many cases there does not appear to be a corresponding reduction in natural water content. In some instances the unconfined compressive strengths show very little tendency to increase with depth, while in all cases field vane determinations do show such a tendency and laboratory vane determinations have a similar though not as pronounced a tendency. One possible explanation of this behavior is found in the fact that in certain of these deposits the amount of varving or silt content appears to increase appreciably towards the bottom of the cohesive stratum. This tendency may be associated with a smaller unconfined compressive strength whereas the field vane strengths might well increase disproportionately because of the presence of this granular material.

Very stiff weathered materials derived from soft clay deposits as a result of exposure to atmospheric agents often prove to be sensitive because of the presence of invisible shrinkage cracks or fissures. These cracks have often been invaded by silt-laden waters which leave behind a very thin coating of cohesionless material. The wall of an excavation made in such weathered material generally will offer very high resistance to any forces that may be applied to it. However, prisms cut from the sides of such excavations are prone to fracture very easily along the irregular shrinkage fissures. The trimming of specimens of regular dimensions from such material is a very difficult and frustrating operation because the material is for the most part very hard and resistant but prone to easily break apart along the shrinkage fissures. Even when a prism of acceptable shape has laboriously been obtained it may prove impossible to determine a realistic figure for its compressive strength because failure is induced by the presence of fissures which

are either open or have been filled by very fine cohesionless silt which forms a mere layer of dust separating two pieces of hard or brittle cohesive material. Lateral restraint such as may be provided in a triaxial compression test will generally lead to more realistic results when testing such a material.

In their natural beds such weathered materials are supported by the surrounding soil and offer much greater resistance to loads than might be inferred from the very low compressive strength which prismatic samples are likely to possess. The use of a form of test which will obviate the need of undisturbed samples in such weathered materials is very great. Consequently, the field vane technique has been applied to determine the properties of these relatively stiff weathered materials. To this end the smallest vane shown in figure 2 is adapted. Frequently rather intense loading can be applied to such firm weathered soils which serve to distribute the pressure so that stresses in the softer underlying soils become moderate. Vane tests made at short vertical intervals provide more reliable and more comprehensive shear strength data than can a more expensive procedure involving undisturbed sampling.

It appears very doubtful that the true strength of sensitive soils can ever be directly measured by small-scale mechanical means other than that represented by a plate bearing test, since no claim can be made that the insertion of a vane does not to some extent disturb and weaken the adjacent soil. The results of vane tests made at the bottom of drill holes appear to merely represent a closer approach to the true strength values than can be expected from results of tests performed upon even the best of so-called undisturbed samples.

It appears that in the case of the materials used as illustrations herein predictions of behavior based wholly upon the results of compression tests would be conservative to an unwarranted degree. In general it would appear that the true strength of the soil may exceed the results computed from unconfined compression tests by as little as 30 per cent or as much as 300 per cent or 400 per cent depending upon the details of the type of soil. It is of course still impossible to state the true strength of such material other than to suggest that it cannot be less than the values revealed by the field vane tests.

#### Implication with Respect to Other Soil Properties

In an analogous manner one may question the determination of the compression characteristics of so-called undisturbed soil samples.

To date a considerable amount of thought has been directed toward the problem of estimating true preconsolidation loads through extrapolation of the results of laboratory tests.<sup>7</sup>

It is believed, and probably with good reason, that if the results of a consolidation test yield a pressure void ratio diagram with a sharp "break" or

7. Casagrande, A., "Determination of the Preconsolidation Load and Its Practical Significance," *Proc., International Conference on Soil Mech. & Foundation Eng'g.*, Vol. III, 1936. Harvard Univ.  
Rutledge, P.C., "Relation of Undisturbed Sampling to Laboratory Testing," *Transactions, ASCE*, Vol. 109, 1944.  
Schmertmann, John H., "Estimating the True Consolidation Behavior of Clay from Laboratory Test Results," *Proc. ASCE* Vol. 79, Paper No. 311, 1953.

change in slope between the recompression and virgin branches of the diagram that the tested sample is reasonably representative of the natural deposit from which it came. However, there is considerable evidence that although the absence of a sharp change in slope in a pressure void ratio diagram appears to be a reliable indication of substantial disturbance of the material, the presence of such a sharp change does not guarantee the integrity of the same. For example, in numerous cases where there existed a pronounced difference between the results of field and laboratory vane tests, consolidation samples gave results that would ordinarily be classed as excellent on the basis of sharpness of curvature in the vicinity of the presumed preconsolidation load. It seems difficult to insist that when laboratory vane test results approximate, for example, but  $3/4$  of the corresponding field vane values, the consolidation test results should be considered highly reliable regardless of how little disturbance occurs during the preparation of the consolidation samples. When the corresponding unconfined compressive strengths approximate only 50 per cent of the field vane strengths it is then evident that trimming of a consolidation specimen is extremely likely to further influence the determination of the preconsolidation load. Furthermore, undisturbed samples secured from fairly uniform deep deposits of sensitive soil often provide well-defined preconsolidation loads which do not vary in any regular fashion with depth.

The results of several consolidation tests made upon samples taken from the boring of figures 3 and 4 indicated that the existing intergranular pressure (caused by overburden) in the layer of blue clay and the underlying varved silt and clay exceeds the preconsolidation loads estimated from the consolidation tests. The excess is rather small at depths less than 50 ft., but amounts to approximately half the overburden pressure at greater depths. This may be entirely due to erratic variation in the actual geological preconsolidation pressure, since there is no data yet available to prove that the deposit is not still consolidating under its own weight. On the other hand, the information shown on figure 4 seems to warrant a belief that the deposit may be naturally consolidated under the existing overburden, because strengths increase regularly with depth. In certain instances, consolidation tests have yielded well-defined preconsolidation loads which were substantially less than the pressures exerted after completion of a structure. However, observations of these structures during and after construction revealed no significant settlement. This would seem to indicate the possibility that the true preconsolidation loads are actually considerably greater than those furnished by the test results even though such results appear to be of excellent quality.

Numerous consolidation tests were made upon samples obtained from the boring detailed in figures 5 and 6. The estimated preconsolidation loads exceed the estimated overburden intergranular pressures from the ground surface to a depth of over 60 ft. beyond which depth preconsolidation pressures vary rather erratically. Above this 60 ft. depth the estimated preconsolidation values all closely approximate  $1\frac{1}{2}$  tons/sq.ft. and the same is true at greater depths, except in the vicinity of the 80 ft. mark where preconsolidation values of 2 and 3 tons/sq.ft. respectively were obtained from two tests. The estimated overburden pressure is 2 tons/sq.ft. at this depth. It is interesting to observe that the values of unconfined compressive strength shown on figure 6 are generally slightly less than 0.3 ton/sq.ft. but that at the 80 ft. depth these values increase to over 0.4 ton/sq.ft., and that at the same depth the laboratory vane results rather closely approximate the field vane results. The increase in the laboratory results at this depth is much greater than the

increase of the field vane results. This appears to constitute strong evidence that the samples taken from this 80 ft. depth were not as sensitive or at any rate were not as much damaged by sampling as those samples taken from somewhat smaller or greater depths. It is difficult to understand why the estimated preconsolidation load to a depth of nearly 70 ft. would show no consistent tendency to vary with depth as contrasted with the variation exhibited by both types of vane shear test unless the estimated magnitudes reflect sample disturbance.

In consideration of the observed differences in shear strength determinations shown herein it is certainly questionable whether any extrapolation of the results of consolidation tests can be guaranteed to furnish reliable data. Relatively little is known about the susceptibility of the apparent preconsolidation load to sample disturbances, as compared with the susceptibility of shear strength to the influence of sample disturbance. Any extrapolation which is not based upon the results of an analysis involving the variables that are pertinent to the phenomenon at hand is fraught with uncertainty. How can such a quantity as "degree of disturbance" be defined in objective, i.e., numerical terms, without knowledge of the properties of a material possessing zero disturbance? Degree of disturbance could be perhaps related to the distortion or deformation of the soil skeleton, but the measurement of such distortion would certainly prove more difficult than measurement of the mechanical properties which it influences. It is suggested that consideration should be given to the development of means for determining preconsolidation loads by testing the natural deposit rather than samples taken therefrom. It is apparent that such testing would in all probability be rather more difficult than the simple vane test described herein. Nevertheless, the development of such a test even if it should prove to be of value only for research purposes would at least provide an indication as to the reliability of any procedure with respect to extrapolating the results of ordinary consolidation tests.

### CONCLUSIONS

The primary purpose of the foregoing remarks is to stimulate interest in the possibilities which appear to reside in the use of simple and relatively inexpensive field tests. It is shown herein that where shear strengths are determined by various techniques those methods which entail the least amount of handling of material tend to give the largest values of strength. The various tests reported herein were performed at comparable rates of deformation. In applying shear strength data to practical problems many other details must be considered, among them

- a) the appropriate rate of shearing strain at which the determinations of strength should be made,
- b) the possibility of progressive failure associated with the weakening which the soil experiences after being subjected to large deformations, and
- c) the ununiform distribution of shearing stresses within the soil mass.

All these factors should be considered of substantial importance in the determination of subsoil stability and each appears to require extensive investigation before it would be possible to specify accurately the appropriate methods by which these factors should be taken into account in design work.

As long as determination of strength is subject to uncertainties attributable to the personal skill used in preparing specimens as well as to the sensitivity

of the particular soil in question the most reliable procedure would appear to be to utilize testing methods which do not require the removal of samples from the ground. The ease with which this can be accomplished is largely dependent upon the nature of the test being performed. It may very well be that the relatively simple technique required to determine shear strength in the field cannot be approached in connection with any other type of testing. Consequently, it may be that a correlation can be established involving laboratory consolidation and shear tests and field shear test results. As long as efforts are restricted to extrapolating laboratory results there must remain doubt as to the accuracy of such extrapolation which can be resolved only by the development of more realistic testing techniques.

## APPENDIX

### Analysis of Loads on Vanes

The vanes are inserted in soil until the four vertical edges are completely embedded. Shearing resistance is assumed to develop on a cylindrical surface described by these vertical edges when the vane is rotated, and also upon a conical surface described by the bottom edges of the vane during rotation. It is assumed that the shearing stress has a constant magnitude over the entire surface of the cylinder and that upon the conical surface the resistance is either a constant and equal in magnitude to that on the cylindrical surface, or else increases linearly from 0 at the axis to a maximum value at the radius of the cylinder, or is in other words proportional to the radial distance. The resisting moment developed on the cylindrical surface is given by:

$$1. \quad M_c = 2\pi R^2 L \tau_{\max} \quad \text{where } \tau_{\max} \text{ is the shear stress.}$$

The resisting moment developed on the conical surface when the shear stress on this surface is constant is given by:

$$2. \quad m = M_c \frac{\sqrt{R^2 + t^2}}{3L}$$

whereas when the stress is proportional to the radial distance the total resistance on the conical surface is:

$$3. \quad m' = M_c \frac{\sqrt{R^2 + t^2}}{4L}$$

The corresponding total soil resistances for these two cases are therefore given by the following two expressions:

$$4. \quad M = M_c \left\{ 1 + \frac{\sqrt{R^2 + t^2}}{3L} \right\} \quad M = M_c \left\{ 1 + \frac{\sqrt{R^2 + t^2}}{4L} \right\}$$

The difference between the two magnitudes given by equations 4 is for practical purposes unimportant. For example, for the shortest vane illustrated in figure 2,

$$\frac{\sqrt{R^2 + t^2}}{L} = \frac{\sqrt{5/4}}{2\frac{1}{2}} = \frac{1}{\sqrt{3}} \quad \text{and so} \quad \frac{1}{3\sqrt{3}} = 0.149$$

$$\text{while } \frac{1}{4\sqrt{3}} = 0.112$$

The difference between these results is therefore less than 3-1/2 per cent of the magnitude of the total resistance given by either of equations 4.

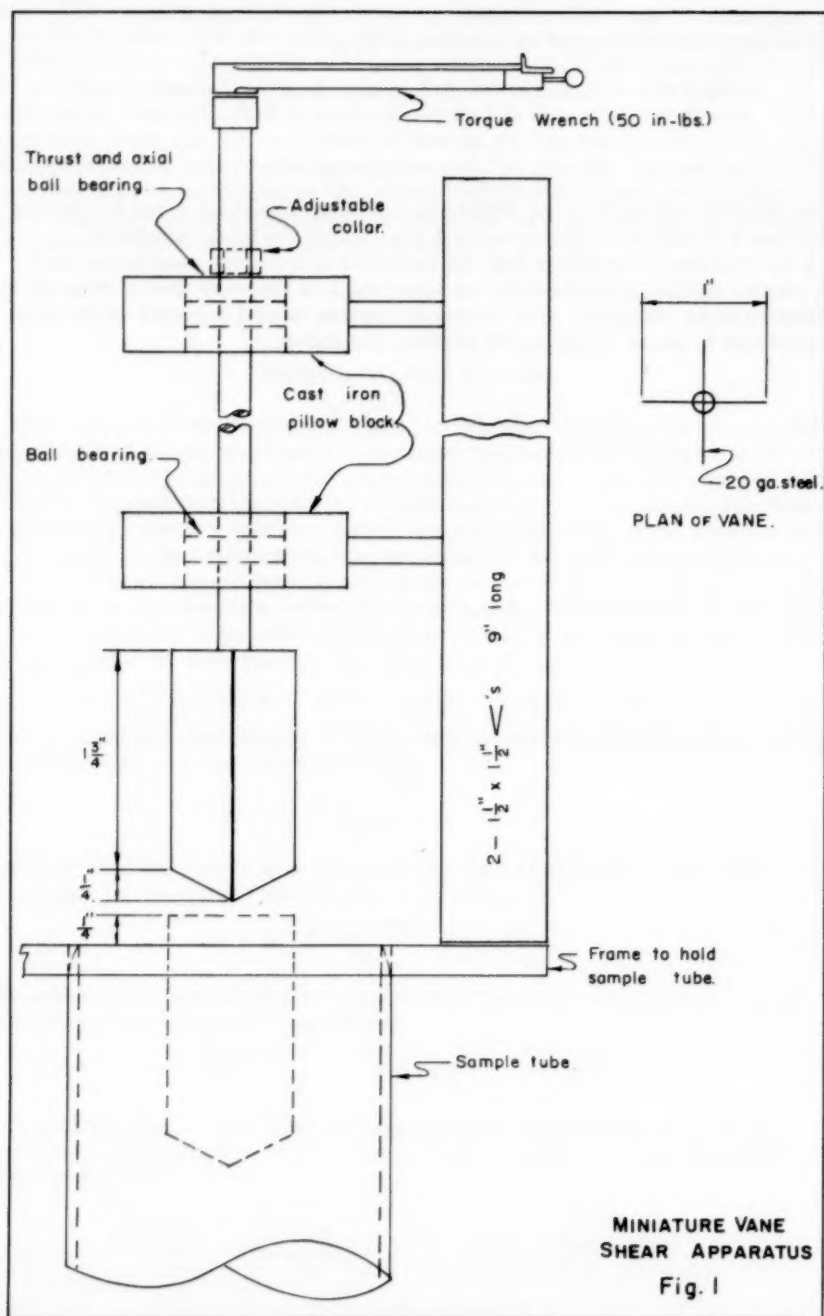
Similarly for the longest vane shown in figure 2,

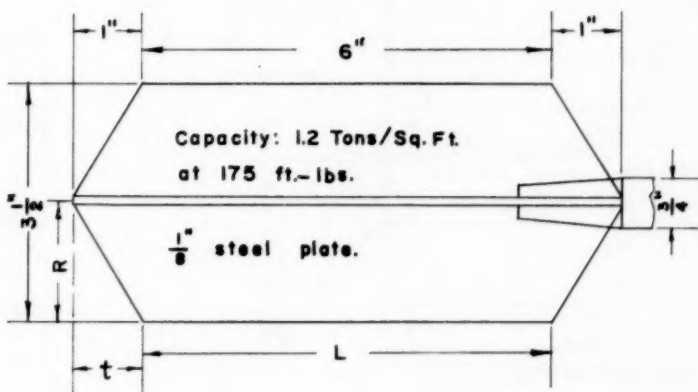
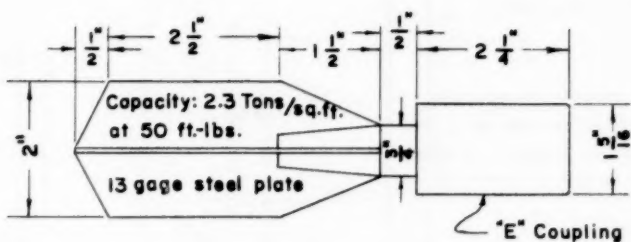
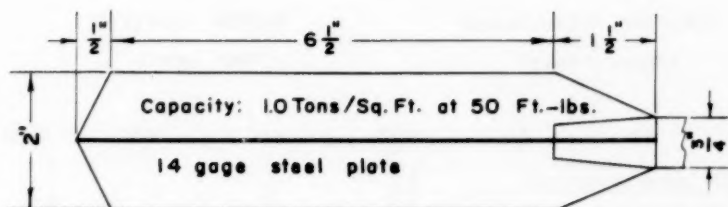
$$\frac{\sqrt{R^2 + t^2}}{L} = \frac{\sqrt{5/4}}{6\frac{1}{2}} = \frac{\sqrt{5}}{13}, \text{ and so } \frac{\sqrt{5}}{3 \times 13} = 0.057, \text{ and}$$

$$\frac{\sqrt{5}}{4 \times 13} = 0.043.$$

from which it follows that the difference between the values given by the two equations #44 will be of the order of 1 per cent of the total resistance.

It is considered therefore that the variation of shearing resistance over the conical surface generated by the lower edge of the vane during rotation is unimportant as compared with natural variations in soil strength which must be expected to occur within short vertical distances.





DETAILS OF VARIOUS FIELD VANES

Fig. 2

(Blades joined by 4 fillet welds)

Fig. 3

PINE POINT BORING No. 4

DRIVING RESISTANCE  
(Blows / Foot)

WATER CONTENT  
(Per cent)

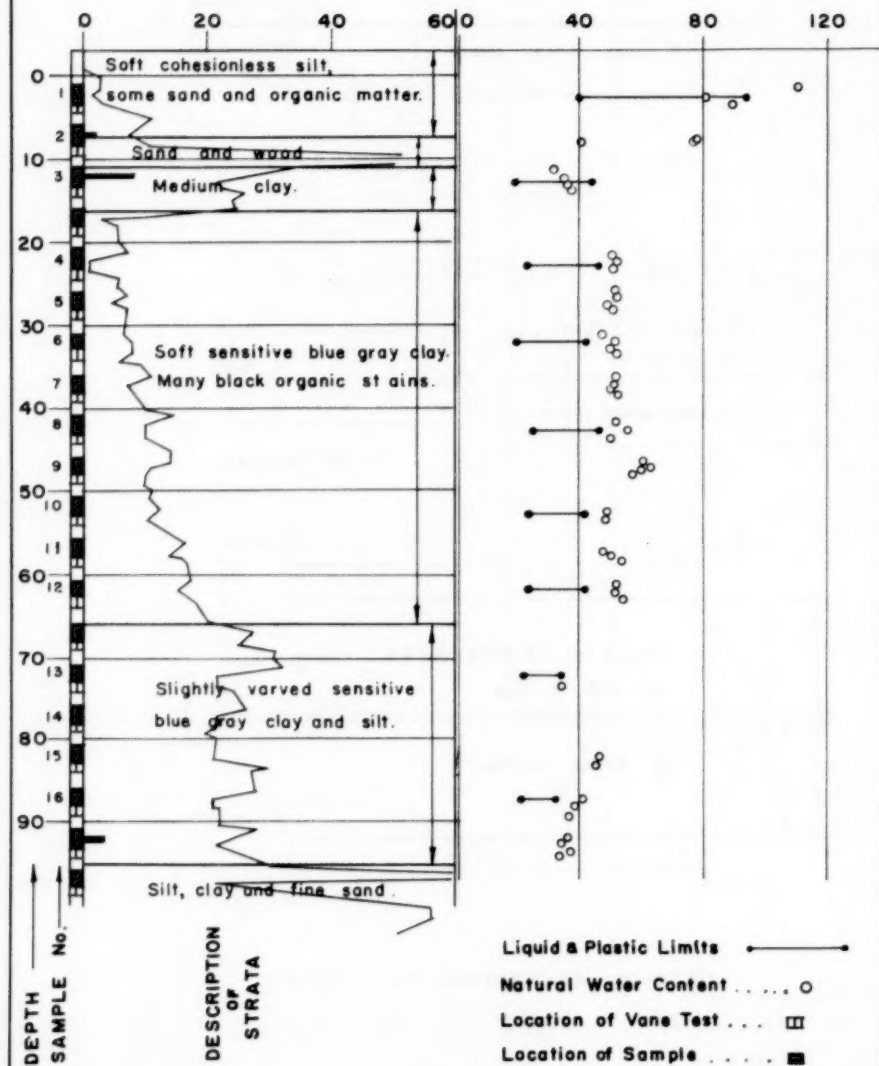


Fig. 4  
PINE POINT BORING No. 4

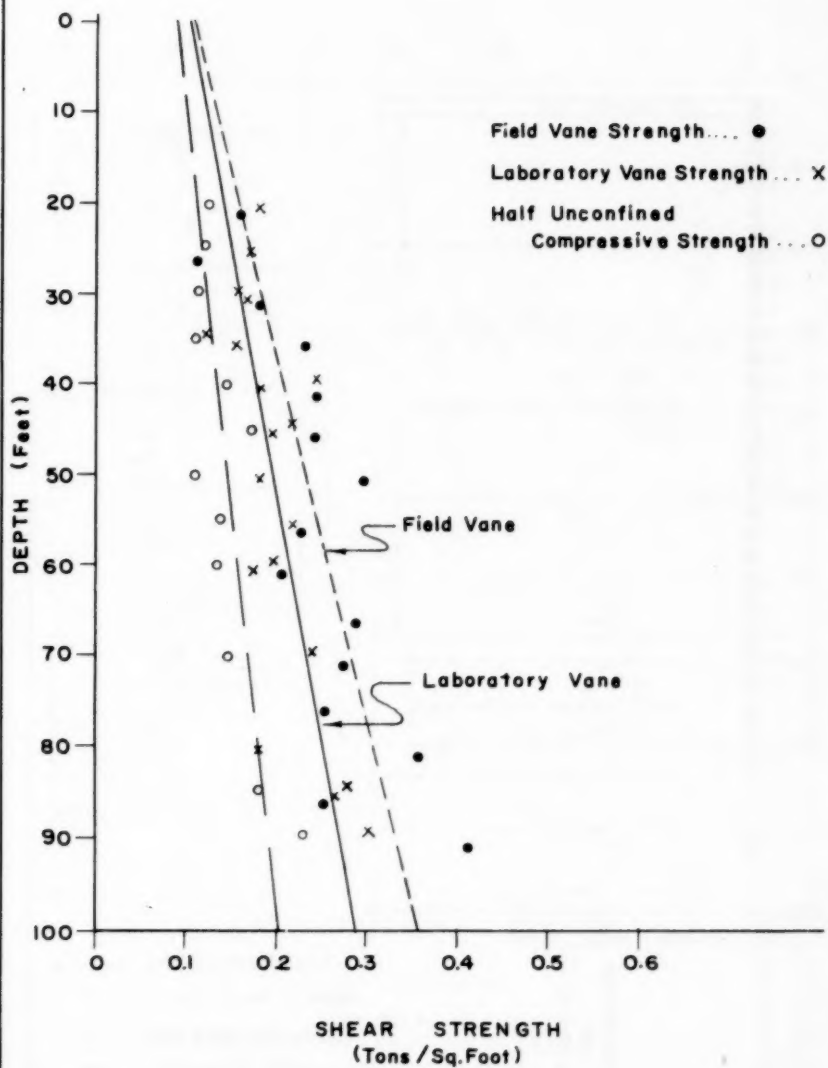


Fig. 5  
MACKWORTH ISLAND BORING No. 4

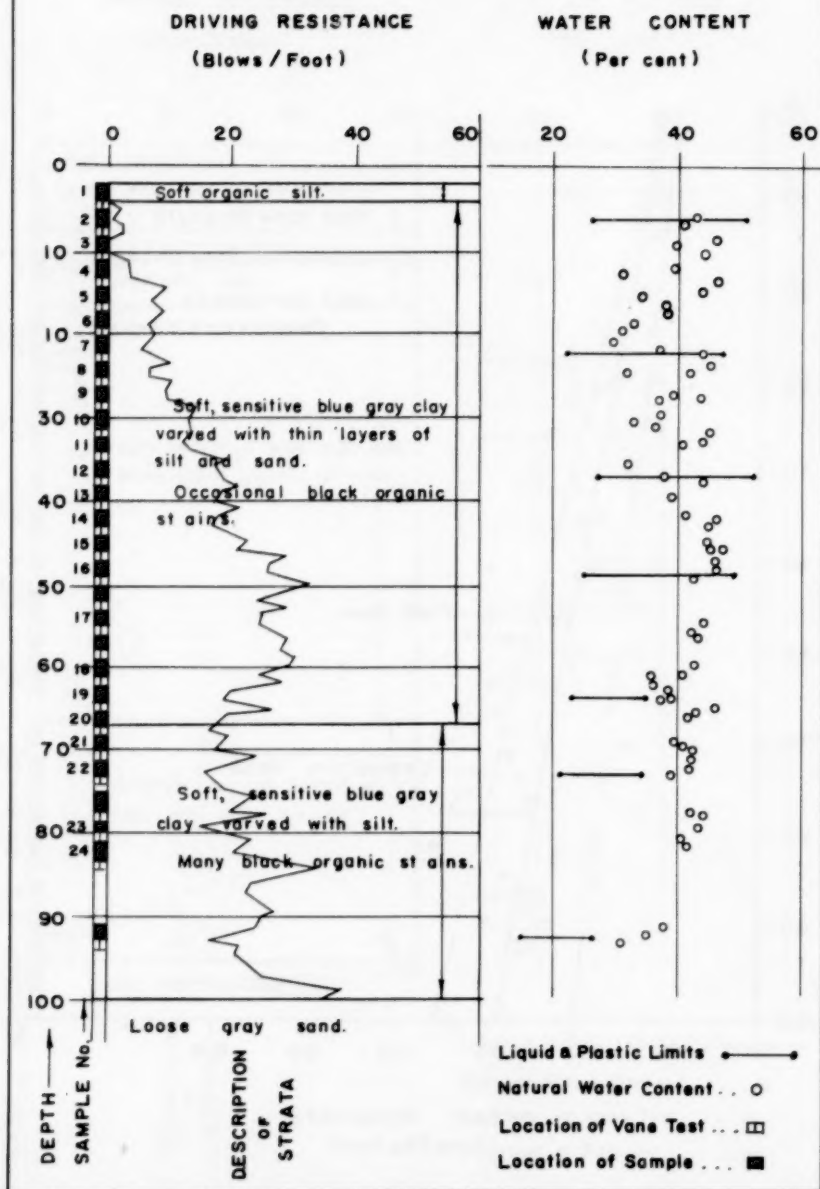
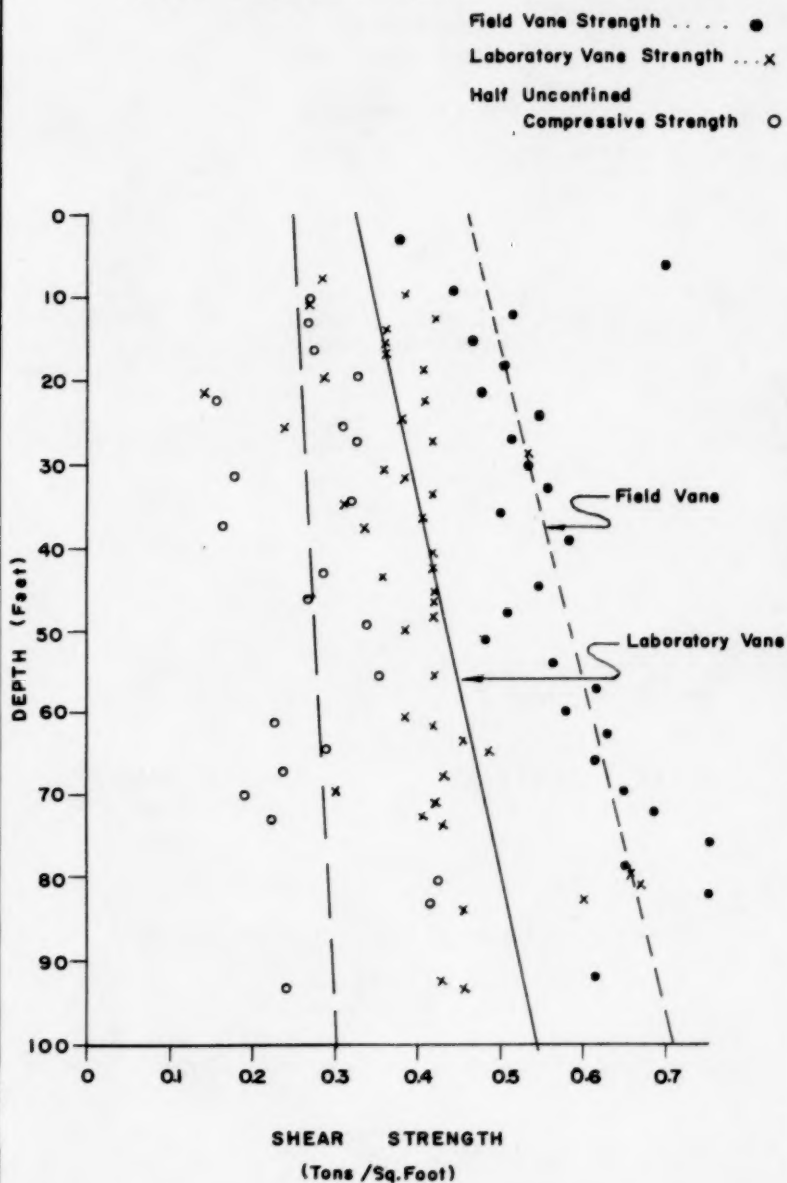


Fig. 6 - MACKWORTH ISLAND BORING No. 4



# AMERICAN SOCIETY OF CIVIL ENGINEERS

## OFFICERS FOR 1955

### PRESIDENT

WILLIAM ROY GLIDDEN

### VICE-PRESIDENTS

*Term expires October, 1955:*

ENOCH R. NEEDLES

MASON G. LOCKWOOD

*Term expires October, 1956:*

FRANK L. WEAVER

LOUIS R. HOWSON

### DIRECTORS

*Term expires October, 1955:*

CHARLES B. MOLINEAUX

MERCEL J. SHELTON

A. A. K. BOOTH

CARL G. PAULSEN

LLOYD D. KNAPP

GLENN W. HOLCOMB

FRANCIS M. DAWSON

*Term expires October, 1956:*

WILLIAM S. LaLONDE, JR.

OLIVER W. HARTWELL

THOMAS C. SHEDD

SAMUEL B. MORRIS

ERNEST W. CARLTON

RAYMOND F. DAWSON

*Term expires October, 1957:*

JEWELL M. GARRELTS

FREDERICK H. PAULSON

GEORGE S. RICHARDSON

DON M. CORBETT

GRAHAM P. WILLOUGHBY

LAWRENCE A. ELSENER

### PAST-PRESIDENTS

*Members of the Board*

WALTER L. HUBER

DANIEL V. TERRELL

---

### EXECUTIVE SECRETARY

WILLIAM H. WISELY

### TREASURER

CHARLES E. TROUT

### ASSISTANT SECRETARY

E. L. CHANDLER

### ASSISTANT TREASURER

CARLTON S. PROCTOR

---

## PROCEEDINGS OF THE SOCIETY

HAROLD T. LARSEN

*Manager of Technical Publications*

DEFOREST A. MATTESON, JR.

*Editor of Technical Publications*

PAUL A. PARISI

*Assoc. Editor of Technical Publications*

---

### COMMITTEE ON PUBLICATIONS

SAMUEL B. MORRIS, *Chairman*

JEWELL M. GARRELTS, *Vice-Chairman*

GLENN W. HOLCOMB

OLIVER W. HARTWELL

ERNEST W. CARLTON

DON M. CORBETT